CAUSES OF THE COLLAPSE OF THE FRONT OF THE MONASTERY OF SAN PELAYO (SPAIN)

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ABSTRACT

At the end of 2010, the collapse of one of the load-bearing walls of the monastery of San Pelayo de Diomondi (Lugo – Spain) occurred. This former benedictine construction, built in the X century, had been declared National Monument in 1931. In particular it was one of the enclosures of the nave attached to the facade of the church of the monastery. At first, the causes of the failure were not at all obvious, due to some defects detected in both the timber structure of the roof and the wall itself. This paper analyzes the construction design, the stresses and displacements of the roof, and the composition of the facade. And according to these data, the causes of the collapse are explained.

Keywords: Collapse, Analysis, Construction

1. INTRODUCTION

On December of 2010, without previous warning, the partial collapse of one of the load-bearing walls of the monastery of San Pelayo de Diomondi, in the province of Lugo (Galicia – Spain) occurred. After the accident, in order to analyze the possible causes that led to the failure of the enclosure, the chief architect in charge of drafting the reconstruction project, considered a priority to examine the possible influence of the structure cover design of the building in the incident, and determine the stresses that this element had on the stone walls of the monument concerned. In subsequent paragraphs first the building and especially its upper enclosure are described. After summarizing the study done on the building design, the recalculation of the wooden structure of the upper enclosure is developed. Finally, the conclusions of the analysis are presented, including some recommendations that perhaps should be considered during the drafting of the reinforcement and consolidation of the building.

2. DESCRIPTION OF THE MONUMENT

Former Benedictine monastery, San Pelayo was raised in the X century, thanks to the donations from the bishops of Lugo, whose bishopric still belongs, with the functions of bishop's residence and palace. The church of the same name was built later, in 1931 was declared National Monument, and currently has parish use only. Fig. 1, taken by the Romanic Circle Association, shows the main facade of the church and the next nave, before the collapse.

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It is interesting to mention that the structure of the affected vessel is mixed, with load bearing walls, consisting of two sheets of granite masonry, with a total thickness varying between 1.20-1.30 m. These walls are formed by rough-hewn stonework, a filling of masonry with different sizes, some of them really big, small stones and clay. As its cover, all the elements of the frame are made of chestnut timer. According to the information in the last structural intervention, was provided for a completely new covering.

3. **DESCRIPTION OF THE ROOFING STRUCTURE**

Structurally the building is covered with a pavilion roof [1], approximately 10 × 10 m. As it is well known, this type of constructive solution was widely used in architecture as upper enclosure for historic buildings with square or polygonal plan. Essentially consists of four rafters of 200 × 250 mm in the cross section, on which a series of joists of 135 × 180 mm are supported, arranged in the direction of the line of maximum slope, spaced at regular distances of 680 mm.

As mentioned above, on top of the rafters is placed a board formed by chestnut tables of 35 mm thickness, on which are arranged directly one waterproof corrugated plate, topped with ceramic curve tile.

To release the space under the roof, the pressure of the rafters is not absorbed with tie beams, but it was necessary to put girders, with 170 × 370 mm section, on the top of the walls. All the frame elements were made of chestnut sawn timber.
It should be mentioned that in this case girders are not constituted by a unique piece of timber, but are formed by two symmetrical elements of about 5.00 m length, joined at the square head by dovetail joints. Fig. 2 shows this type of joint, which in essence is a connection unable to withstand the stresses of the rafters and the joists, while not allowing a certain displacement in the horizontal plane.

In the same image it can also be appreciated that the support of the rafters to the frame has been materialized through a carpentry joint, known as bird mouth.

Fig. 3 shows this structure, including load-bearing walls and the area where the collapse is located.

4. COLLAPSE PROCESS

On December 19, 2010 the bearing wall located on the East side of the church, collapsed. The incident affected mainly the central enclosure, and allowed to discover an old window, which had been walled up by stones and mortar.

When the collapse was known, in order to strengthen the affected area, it was ordered the application of a hydraulic lime mortar, ensure the gaps by timber and steel frame crosses (Fig. 4), and provide a security support for the affected corner (Fig. 5).

As can be seen, the affected area was located just below the dovetail joint. Hence it was initially considered this factor as a possible origin of the collapse process.

Fig. 3 Plant of the building and roof cross section

Fig. 4 Reinforcement crosses and application of lime mortar
5. INVESTIGATION AND FIELD SURVEY

Once the stability of the walls was assured, the first survey of the building was developed. From the simple visual analysis of the condition of the structure after the failure, some conclusions could be issued:

1) The composition of the load-bearing walls was poor, with stones which sections were almost circular or elliptical, and large size. In addition, the mortar used lots of clay, which further complicated the stability of the facade.
2) The lintels of the openings, formed by wooden joists, were affected by decay, due to ancient attacks of fungus and insects, and also had an excessive moisture content.
3) And above all, as it was said before, the timber roof presented a major design mistake on the conception of its wooden frame. Indeed, had it been properly defined this element of construction, the symmetry of this type of roof allows an easy compensation of the efforts, so that their mechanical behavior never supposes a problem of stability.

However, in this case, the existence of a dovetail assemble in the middle of the girders, probably influenced the collapse: actually this joint allows the transmission of the tensile stresses to the top of the bearing walls; and of course, it causes the displacements of the roof frame.

In fact, during the survey, a relative displacement close to 200 mm of the bearing wall was measured, which seemed to justify the first impression of the existence of unabsorbed horizontal thrusts at the level of the girder.

6. PATHOLOGICAL STUDY

Considering these design mistakes and the structural damage, the best way to confirm the preceding hypothesis which would explain the collapse of the wall, would be to perform a structural analysis of the cover. This could quantify the stresses transmitted by the timber frame to the wall, and then justify the claim.

Thus, from the field data, the design and the dimensions of the scantlings, a cover model was performed (Fig. 6).

As for the structure, calculation was performed according to the requirements contained in the existing Spanish Technical Building Code, (CTE) [2], first published in March 2006 and revised in April 2009, and assuming a Strength Class D35 for chestnut wood.

It is important to note that the connections and joints between diagonal tie beams and the girders were supposed to be perfect fixities, but in reality they would not, given that almost surely produce some
local displacement at these points, which have a direct impact on the joint [3]. Note that these diagonal tie beams limited the actual length of the girder to 215 cm, so that the maximum displacement at this point could never exceed the value of 14 mm.

**Fig. 6** Modeling of the timber frame

And certainly calculation checks performed confirmed this pre-diagnosis because, for a design such as that provided with dovetail joints in the girders, the deformations exceeded these maximum allowed. Moreover, regardless of the significant stresses occurred in the dovetails, the measured displacement at these points was 27.36 mm (Fig. 7). That is, a deformation of L/78, that it would be absolutely inadmissible. However, these results were far from the 200 mm recorded in the tower.

**Fig. 7** Maximum displacement of the dovetail joint

Therefore, the only way to justify a deformation of this magnitude would be add the ones due to the assemblies diagonal beam – sleeper. With them, the girder himself reached a deformation next to the 145 mm exactly at the joint. And consequently, its thrust on the walls could have produced a displacement of more than 200 mm in the coronation of the wall.

These results seemed to point that the facade, consisting of large bowls, locked with a clay mortar, and weakened by the penetration of rainwater through the top of the bearing walls, was not able to withstand a tensile stress as high as the one transmitted by the roof, and collapsed. Hence, it is noted a very important deformation in the contact surface girder – wall.

In any case, it must be remembered that there were no reliable data on the type of connection defined for assemblages between girders, nor on the possible injury, damage or even presence of features such as knots, ring shakes, slope of grain, etc. existing in other joints. And much less, if a defect was detected during the construction of the building.

In other words, the structural testing of this roof was modeled on the side of safety, assuming a complete absence of mistakes, defects or damages in the timber pieces. However, knowledge of these data is essential in order to design the reinforcement solution.
7. CONCLUSIONS

According to the information collected on site and the results of the structural analysis, with the current configuration, it can be concluded that the collapse of the wall was due to a combination of two factors:

1) First, the thrust of the roof frame, specially the girders, due to the incorrect design of the timber frame. Indeed, through the structural check has been possible to verify that the bending moments of the rafters and the girders were excessive, causing too high deformations at the top of the wall, affecting its stability.

2) And especially the poor construction and maintenance of the load bearing wall, consisting of big stones, some with nearly circular section, stuck with a clay mortar, which was further degraded by the action of water.

8. RECOMMENDATIONS

In principle, the provision of a dovetail joint at the square end of the girders that make up the perimeter tie set, seems to have decisively influenced the development of the collapse process. Indeed, it was found that the girders, with an assemblage at the center of its length, have not been able to absorb the thrust of the rafters, transmitting an over-stress to a wall already weakened by its own constitution and also the action of the water, which caused its ruin. Therefore, only as far as the roof structure is concerned, as long as its components have not undergone any cracks or other damage that obliges replacement or reinforcement, it is considered that it should be performed at least two types of intervention. The proposed actions are the following:

1) To stitch all the dovetail joints by the provision of steel bars, glued to the girders.
2) To place steel fittings at the ends of the diagonal braces, to ensure the embedding of the joint. If possible, we recommend the use of forks to embrace the sleepers from the outside, fixed to the upper and lower diagonal braces thereof, and with no connection with the girders. We do not recommend the use of bonded bars at these points, as they prevent swelling / shrinkage inherent in the wood, which could cause some kind of break in the timber elements.

3) Prior to the implementation of the solution that finally was decided, it is important to emphasize the need to study in detail the type of connection between girders, and between girders and diagonal braces, and the desirability of reviewing the roof structure, for the absence injury, damage or defects existing in the timber frame.

4) And in general, for any type of construction, it is recalled that the conception stage of the structure is undoubtedly the most important, and directly dependent on the durability of the building. In the current Spanish Technical Building Code, (CTE), and also in the EC5 [4], special attention is paid to protection by design, and its relation to the damages caused by rain water. But do not forget that the deformations and bracing are really two factors that most influence immediately on the durability of a structure, and will certainly influence their design and dimensioning of timber frames, especially on roofs structures.

REFERENCES